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Ultimate strength of rolled and welded columns and beam-columns /

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ULTIMATE STRENGTH OF ROLLED AND WELDED COLUMNS AND BEAM-COLUMNS

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ABSTRACT

An analysis of the ultimate strength of steel columns and beam-columns manufactured with different processes is carried out. Particular emphasis is placed on residual stresses as a principal cause for the variation in the ultimate strength. Since the purpose of the study is to examine the load-carrying capacity of column members in typical multistory building frames, a maximum slenderness ratio of 60 is assumed.

A comparison of the available test results of centrally loaded columns with predictions based on the recently proposed multiple column formulas is presented. A majority of the test results is found to fall in a relatively narrow band.

Ultimate strength analysis is performed on rolled and welded beam-columns using the column deflection curve concept. The residual stress distributions assumed in the analysis are those considered to be typical in rolled shapes, welded shapes with sheared plates, welded shapes with flame-cut plates, and annealed shapes.

The results show that for major axis bending welded members are likely to be stronger than rolled members because of the more favorable residual stress distribution.

1. INTRODUCTION

This thesis is concerned with the determination of the load-carrying capacity of columns and beam-columns in the inelastice range with slenderness ratio up to sixty. This value is considered to be a maximum for slenderness ratios of columns in typical multistory building frames.

1.1 Preview Research

Much work has been done to study the strength of centrally-loaded columns in the elastic and inelastic range. All developed methods have advantages and drawbacks when they are applied to determine the actual behavior of a column. The first work was done by Euler,[1] now recognized as a classical theory, and published in 1759. Euler's work was followed by Engesser in 1889,[2] who proposed the tangent modulus concept for analyzing inelastic column buckling. In 1895 he also proposed the reduced modulus (double modulus) approach after considerable criticism on the tangent modulus concept by Considere.[3] The latter theory was supported by Karman through tests.

The tangent modulus theory was generally accepted because test results showed to be closer to it rather than to the reduced modulus theory. Shanley was the first one to give a complete explanation of the inelastic column buckling phenomenon. He also showed that the reduced modulus load is an upper bound and the tangent modulus load a lower bound to the column strength. All the studies were made on initially straight columns and dealt primarily with the initiation of buckling (bifurcation of equilibrium). The behavior of columns when subjected to loads above the tangent modulus load has been studied by by Duberg and Wilder,[5] and by Johnston.[6] It is now possible, by analytical means, to determine the post-buckling strength (or ultimate strength) of a column with or without initial out-of-straightness.

The influence of residual stresses on column strength was first explained by Osgodd[7] and by Yang et al.[8] An extensive investiation on the inelastic buckling strength of steel columns with residual stresses was subsequently carried out at the Fritz Engineering Laboratory. This study included both rolled and welded shapes and also high strength steel columns. A summary of the results of this investigation is given in Reference 9. It has been found that the residual stress distribution in a member is influenced by the following factors:

1. Grade of steel
2. Manufacturing method (rolling vs. welding, flame cut plates vs. sheared plates)
3. Cross-sectional shape (wide-flange, box, etc.)
4. Size of shape and thickness of the component plates
5. Magnitude and shape of out-of-straightness

These factors in turn affect the inelastic buckling strength as well as the ultimate strength of the member. For shapes with major and minor axes (such as wide-flange), the axis of buckling also becomes an important factor.

Bjorhovde made an extensive probabilistic analysis of the influence of these factors and proposed two sets of multiple column curves for a variety of wide-flange and box shapes.[9] One set of the multiple column curves is based on the tangent modulus load, and the other is based on the ultimate strength with an assumed initial out-of-straightness of 1/1000. The use of the multiple column curve concept in practical design is being examined at the present time.

The strength of steel beam-columns has been studied by numerous investigators for several decades.[10] The work on residual stress as described above has been extended to beam-columns. The results indicate that residual stress also has a significant effect on the moment-carrying of a beam-column.[11,12] Therefore, the strength of a beam-column is also affected by all the factors mentioned above. A thorough examination of these factors has not yet been made.

1.2 Objective and Scope of Study

The main objective of the study is to evaluate the strength of rolled and welded columns and beam-columns with low slenderness ratios (lower than 60). Only failure due to flexural buckling is considered.

The thesis consists of two major parts:

1. A comparison of all the available test results of centrally loaded columns with predictions provided by the multiple column curves based on ultimate strength.
2. Ultimate strength analysis of rolled and welded beam-columns. Numerical results are obtained for a beam-column whose slenderness ratio is equal to 40.

2. RESIDUAL STRESS DISTRIBUTION AND INFLUENCE OF FABRICATION

The distribution and magnitude of the residual stresses in a member are influenced by the method used in its manufacture. Idealized patterns of residual stress distributions, based on experimental measurements, of rolled and welded shapes are shown in Figure 1.[13] For the flanges and webs of rolled wide-flange shapes, triangular patterns of residual stress distribution are assumed, in which the magnitudes of the maximum compressive and tensile residual stresses are equal. For rolled shapes of A36 steel and without heat treatment the maximum values are assumed to be $0.3\sigma_y$. For heat-treated members, these values are reduced to $0.1\sigma_y$. It has been previously reported that rolled shapes exhibit the least reduction in column strength due to residual stress.[14]

For welded H shapes the maximum tensile residual stress occurs near the welds and is assumed to be equal to the yield stress of the material. If the component plates are flame cut, tensile residual stress also occurs at the tip of flanges. The presence of these tensile stresses significantly improves the strength characteristics.

A detailed discussion of these idealized residual stress patterns is given in Refence 13.

3. ULTIMATE STRENGTH OF COLUMNS

In this section, an analysis of the available test results on centrally loaded column with slenderness ratio less than 60 is made. All the results are compared with predictions based on the multiple column curves proposed by Bjorhovde. The ultimate strength column curves are used as the basis for comparison.

3.1 Available Test Results

Column tests have been conducted on specimens made of various materials. For columns with slenderness ratios less than sixty, the following steels have been used in making the test specimens:[9]

A7 steel with σ_y = 33 ksi (five tests)

A36 steel with σ_y = 36 ksi (nine tests)

A242 steel with σ_y = 50 ksi (one test)

A572 steel with σ_y = 50 ksi (two tests)

A514 steel with σ_y = 100 ksi (nine tests)

The test specimens had different cross-sectional shapes, wide-flange, H, and box, and were manufactured by rolling and welding. They also included light and heavy shapes. (A shape is defined as light if the thickness of all component plates in less than one inch.[10])

Summaries of the cross-sectional and material properties of all the test specimens together with the experimental results are given in Tables 1, 2, and 3. Also shown in the tables are theoretically calculated ultimate maximum load and empirical predictions based on the applicable column curves or formulas proposed by Bjorhovde.[9]

3.2 Maximum Strength Column Curves

Using computer simulation, Bjorhovde developed 112 column curves relating the maximum load to the column slenderness ratio. The non-dimensional parameters used are $\frac{P_{max}}{P_y}$ and $\lambda = \frac{1}{\pi} \sqrt{\frac{\sigma_y}{E}} \left(\frac{L}{r}\right)$, the latter being referred to as "slenderness function".

The proposed curves were assembled in three groups, according to the maximum load, material properties, and manufacturing method. The choice of three groups, and hence three column strength curves, was made on the basis that advantage may be taken of the strength of the stronger columns without complicating the design procedure.

The accepted number of curves appeared to be satisfactory for medium and high slenderness ratios. For columns with low slenderness ratios, the necessity of working with three different curves in design calculations has been questioned. It would appear that two curves may give satisfactory results. That is due to the fact that for low slenderness ratio columns the variation of the maximum strength is dependent primarily on the variation of the yield stress rather than on any other factors. The pronounced influence of the yield stress of the column material contracts the band width of the maximum column curves with a reduced mean value associated with each curve.

The three column curves proposed by Bjorhovde are:

Curve 1 $$\frac{P_{max}}{P_y} = 0.99 + 0.122\lambda - 0.38\lambda^2 \quad \text{for} \quad 0.15 \leq \lambda \leq 1.2$$

Curve 2 $$\frac{P_{max}}{P_y} = 1.035 - 0.204\lambda - 0.232\lambda^2 \quad \text{for} \quad 0.15 \leq \lambda \leq 1.0$$

Curve 3 $$\frac{P_{max}}{P_y} = 1.095 - 0.63\lambda \quad \text{for} \quad 0.15 \leq \lambda \leq 0.8$$

3.3 Comparison of Test Results with the Proposed Column Curves

The available test results are compared with the proposed column formulas which were developed for an initial out-of-straightness $e/L = 1/1000$. Figures 2 through 6 show that the test points, except one Japanese test, belong to the bands assigned to Curves 1 and 2.

The comparison given in Tables 1, 2 and 3 is made for the loads obtained from the tests, the theoretical loads, and the loads predicted by the proposed column formulas. The ratios between the theoretical and the experimental loads lie within 1 ± 0.05, and they apparently satisfy the required accuracy. An average of the absolute difference $\delta = 4.9\%$ is obtained for all the tests. Also given in the tables are the ratios between the experimental loads and the empirical predictions. In this case greater differences are found with an average absolute difference of $\alpha_1 = 5.4\%$. The larger differences occur near the maximum slenderness ratios adopted for the analysis.

With respect to group distribution of variously manufactured columns, it is found that:

Group 1 includes: rolled wide-flange made of A514 steel, rolled box shapes, annealed columns, welded box of A514 steel, and welded wide-flange of A514.

Group 2 includes: rolled wide-flange shapes of A36 and A242 steel, welded box made of A7 steel, and welded wide-flange manufactured with flame-cutting of all applied materials except A514 steel.

4. ULTIMATE STRENGTH OF BEAM-COLUMNS

Several analytical methods are availbale for determining the response of beam-columns in the elastic and inelastic range. These methods can be used to obtain the combination of axial force and bending moment which causes failure of a given beam-column. The response of a beam-column may be studied by relating the applied end moment, M_o, to the end rotation θ_o, for the entire history of loading. For the convenience of calculation and interpretation of results, it is customary to keep the axial force, P, applied to the member constant and gradually increase the end moment. The peak of the end moment vs. end rotation curve determines the moment-carrying capacity. In this study, the moment-carrying capacity is determined for beam-columns subjected to end moments causing symmetrical single curvature deformation. The bending moments are applied about the major axis of a rolled wide-flange or a welded H shape.

4.1 Assumptions

The following assumptions are made in the analysis:

1. The stress-strain properties of the column material are elastic and perfectly plastic, and the effect of strain hardening is neglected.
2. For a given combination of axial force and bending moment acting at a section, there exists a unique value of curvature. This means that the deformation of a section depends only on the final values of the axial

force and bending moment, and that the actual history of loading does not affect the resulting curvature.

3. The effect of shear is small and can be neglected.

4. Weak-axis buckling and lateral-torsional buckling are effectively prevented so that failure is always caused by excessive bending in the plane of the applied moment.

4.2 Moment-Thrust-Curvature Relationships

The basic information that is necessary in performing a response analysis of a beam-column is the moment-curvature-thrust relationships (M-P-ϕ) of the member's cross section. The M-P-ϕ relationships used in the analysis were determined for the W8 x 31 section by a separate program. In this program a moment vs. curvature curve was developed for a constant axial thrust by dividing the cross section into a large number of finite elements (Figure 7). The strains of the elements are related to the curvature of the section and stresses to the applied bending moment. The relationship between the applied moment and the resulting curvature can therefore be found through equilibrium and compatibility conditions of these elements. The details of the method and the computer program are described in Reference 15.

The basic program can be easily modified to take into account the effect of residual stresses. The distribution and magnitude of the residual stresses assumed in the analysis are those given in Figure 1. To study the effect of residual stresses on the moment-thrust-curvature relationships, computations were carried out for the W8 x 31 shape manufactured by the following processes:

1. Rolling
2. Welding with sheared plates
3. Welding with flame cut plates
4. Annealing after rolling or weldiing (free of residual stress)

Numerical results are obtained for three axial force values, $P=0.4P_y$, $0.6P_y$ and $0.7P_y$ where P_y is the axial yield load. The M-ϕ curves for $P=0.4P_y$ and $0.6P_y$ are shown in Figure 8. The curves for the welded shape with sheared plates and with flame-cut plates are too close to differentiate. Also, these curves are almost identical to the M-ϕ curve of the annealed shape. In Figure 8 the bending moment, M, is plotted as the ratio M/M_{pc} where M_{pc} is the full plastic moment corresponding to the specified value of P. For example, the M_{pc} value corresponding to $P=0.4P_y$ is used to non-dimensionalize the M values for the curves marked $P/P_y=0.4$. The curvature ϕ is also expressed non-dimensionally as the ratio ϕ/ϕ_{pc} where $\phi_{pc} = M_{pc}/EI$.

When the M-P-ϕ relationships thus obtained are used in the beam-column analysis, the final results will permit the evaluation of the influence of variation in residual stress on the strength of beam-columns. The M-P-ϕ relationships given in Figure 8 indicate that in the inelastic range the welded shape and the annealed shape are stiffer than the rolled shape. A stiffer member usually will deflect less and consequently will have less pronounced instability effect.

4.3 End Moment vs. End Rotation Relationships and Ultimate Strength

The column deflection curve (CDC) concept is used to construct the end moment vs. end rotation relationships of the beam-columns selected for this study. This concept is fully explained in Reference 16. The slenderness ratio (L/r_x) of the beam-columns selected is 40 and the axial forces are equal to $P=0.4P_y$, $0.6P_y$ and $0.7P_y$. The moment-rotation curves of the beam-columns are shown in Figures 9, 10 and 11 for the three axial force values. It is found that in the inelastic range the welded beam-columns are indeed stiffer than the rolled beam-columns (less end roation and transverse deflection throughout the members). The moment-carrying capacity of the welded members is higher than the rolled members. The difference is about 3.5% for $P/P_y=0.4$, 10% for $P/P_y=0.6$, and 1% for $P/P_y=0.7$.

Since the M-P-ϕ curves of the welded shape with sheared plates and with flame-cut plates are identical to those of the annealed shape, the moment-carrying capacities of the welded and annealed columns are also identical. This implies that residual stresses have no influence on the strength of the beam-columns studied.

5. CONCLUSIONS

On the basis of the results presented in this thesis, the following conclusions can be made, concerning the strength of columns and beam-columns with a maximum slenderness ratio up to 60:

1. In the case of centrally loaded columns, the third column curve can be avoided for practical applications. This conclusion must be viewed with caution because the available test data for low slenderness ratio columns are very limited.
2. Within the range of slenderness ratio included in the study, the average difference between the experimental column loads and the predicted loads based on the proposed curves is about 5.4%.
3. In the case of beam-columns bent about the major axis, welded columns are found to be stronger than rolled columns. The maximum difference in the moment capacity is about 10%.
4. The strength of welded columns with sheared plates and with flame-cut plates is almost identical to that of the annealed columns. Residual stress appears to have no effect on the moment-carrying capacity.

6. TABLES

TABLE 1

Comparison of Theoretical and Experimental Maximum Column Strength for Rolled Wide-Flange Columns

Shape	Steel Grade	Light or Heavy	Axis	Experiment				Theory		Empirical	
				e/L	λ	L/r	Pmax/Py	Pmax/Py	α	Pmax/Py	α_1
W12x161	A36	H	W	.002	0.49	50	0.78	0.83	1.06	0.88	1.13
W8x31	A242	L	W	.0009	0.75	54	0.82			0.75	0.91
W12x120	A514	H	W	? .0002	0.55 0.92	30 50	0.89 0.82			0.94 0.78	1.05 0.95
W10x112	A514	H	W	.0001	1.07	49	0.73			0.69	0.95

TABLE 2

Comparison of Theoretical and Experimental Maximum Column Strength for Welded Wide Flange Columns

Shape	Steel Grade	Light or Heavy	Axis	Experiment				Theory	α	Empirical	
				e/L	λ	L/r	Pmax/Py	Pmax/Py		Pmax/Py	α_1
H12x79	A36	L	W	.0002 .0003	0.35 0.70	30 60	0.97 0.76	0.93 0.75	0.96 0.99	0.99 0.78	1.02 1.03
H14x202	A36	H	W	.0009 .0006	0.34 0.68	30 60	0.97 0.84	0.94 0.78	0.97 0.93	0.99 0.79	1.02 0.94
H12x79	A572	L	W	.0003 .0011	0.40 0.81	30 60	0.90 0.76	0.91 0.70	1.01 0.92	0.92 0.78	1.02 1.03
H14x202	A572	H	W	.0006	0.84	60	0.80	0.70	0.88	0.82	1.02
H10x62	A514	L	W	.0004 .0003	0.68 1.07	35 55	0.90 0.79	0.90 0.83	1.0 1.05	0.9 0.69	1.0 0.87

T A B L E 3

Comparison of Experimental and Theoretical Results for Welded Box Columns

Shape	Steel Grade	Light or Heavy	Axis	Experiments e/L	Experiments λ	Experiments L/r	Experiments Pmax/Py	Theory Pmax/Py	α	Empirical Pmax/Py	Empirical α_1
□6x20	A7	L	P	.0006 .0002	0.44 0.70	32 51	0.93 0.75	0.90 0.73	0.97 0.97	0.90 0.78	0.97 1.03
□10x65	A7	L	P	.0003 .0006	0.34 0.56	30. 50.	0.94 0.82	0.96 0.85	1.02 1.04	0.94 0.85	1.0 1.04
□6x20	A514	L	P	.0007 .001	0.76 1.15	40. 60.	0.91 0.69	0.85 0.63	0.94 0.91	0.86 0.63	0.95 0.91
□10x65	A514	L	P	.0001 .0005	0.56 0.94	30. 50.	0.94 0.87	0.91 0.82	0.97 0.94	0.90 0.77	0.96 0.89

7. FIGURES

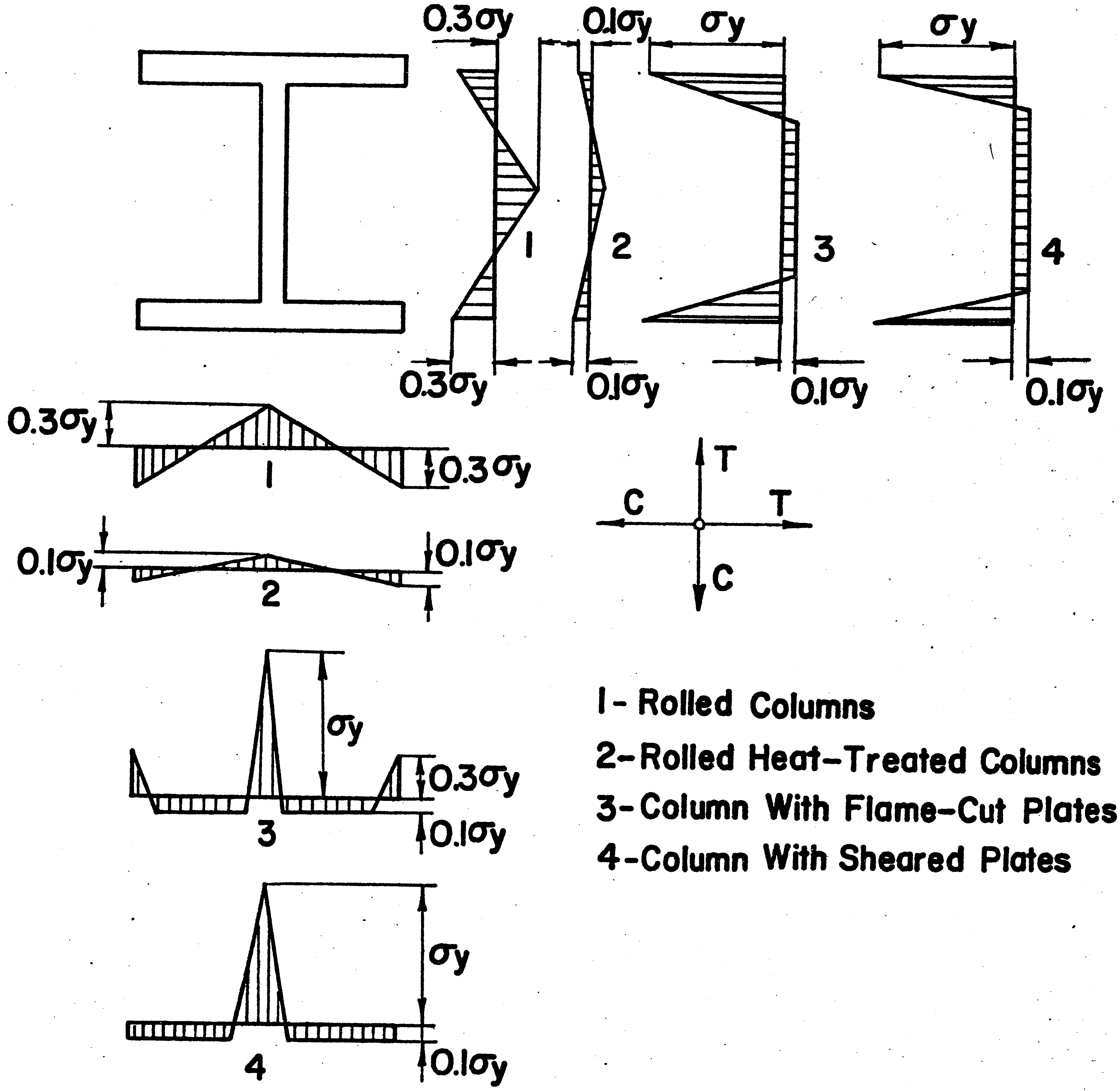

Fig. 1 Residual Stress Distribution for Various Manufactured Columns

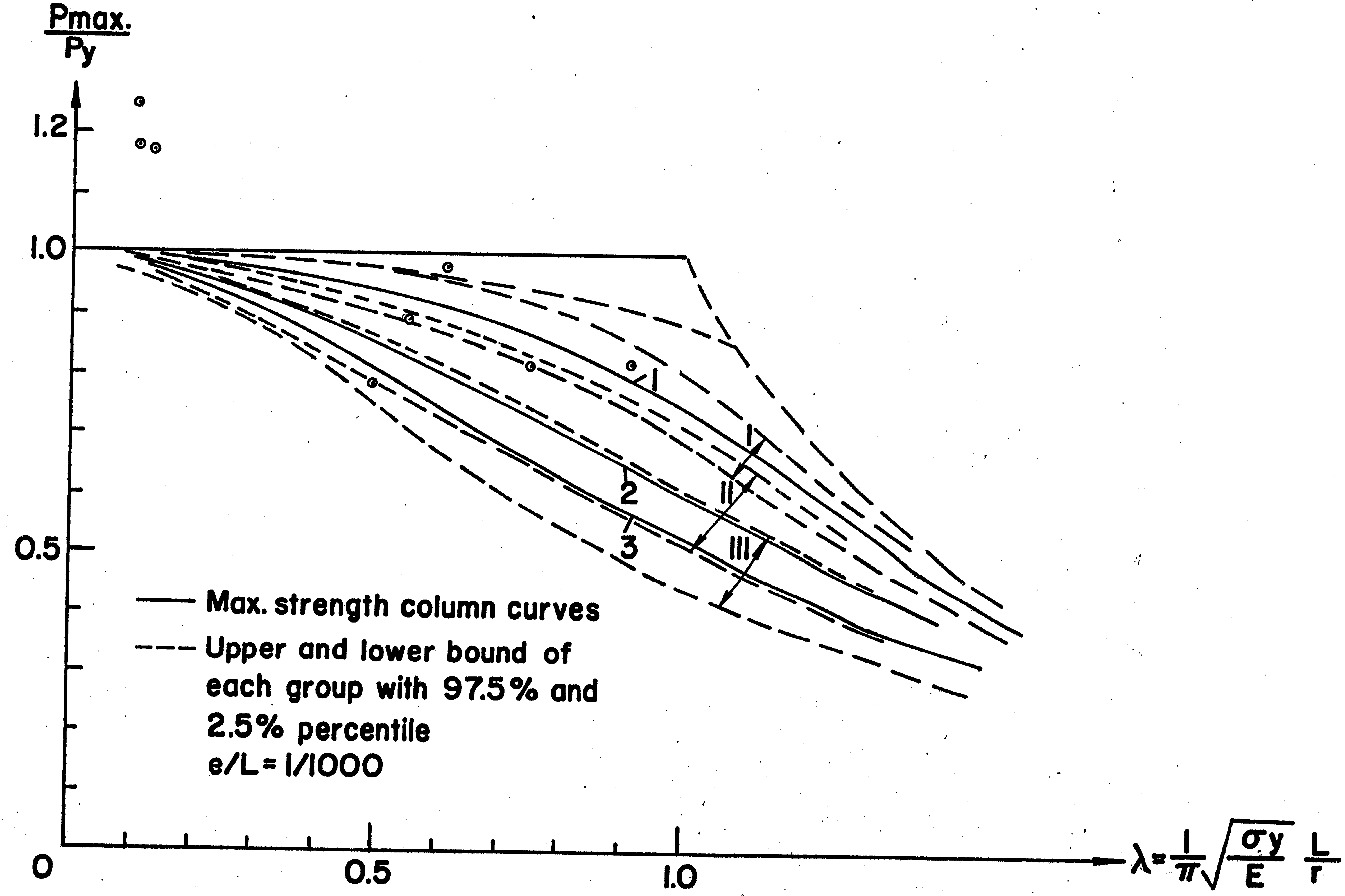

Fig. 2 Test points for rolled wide flange

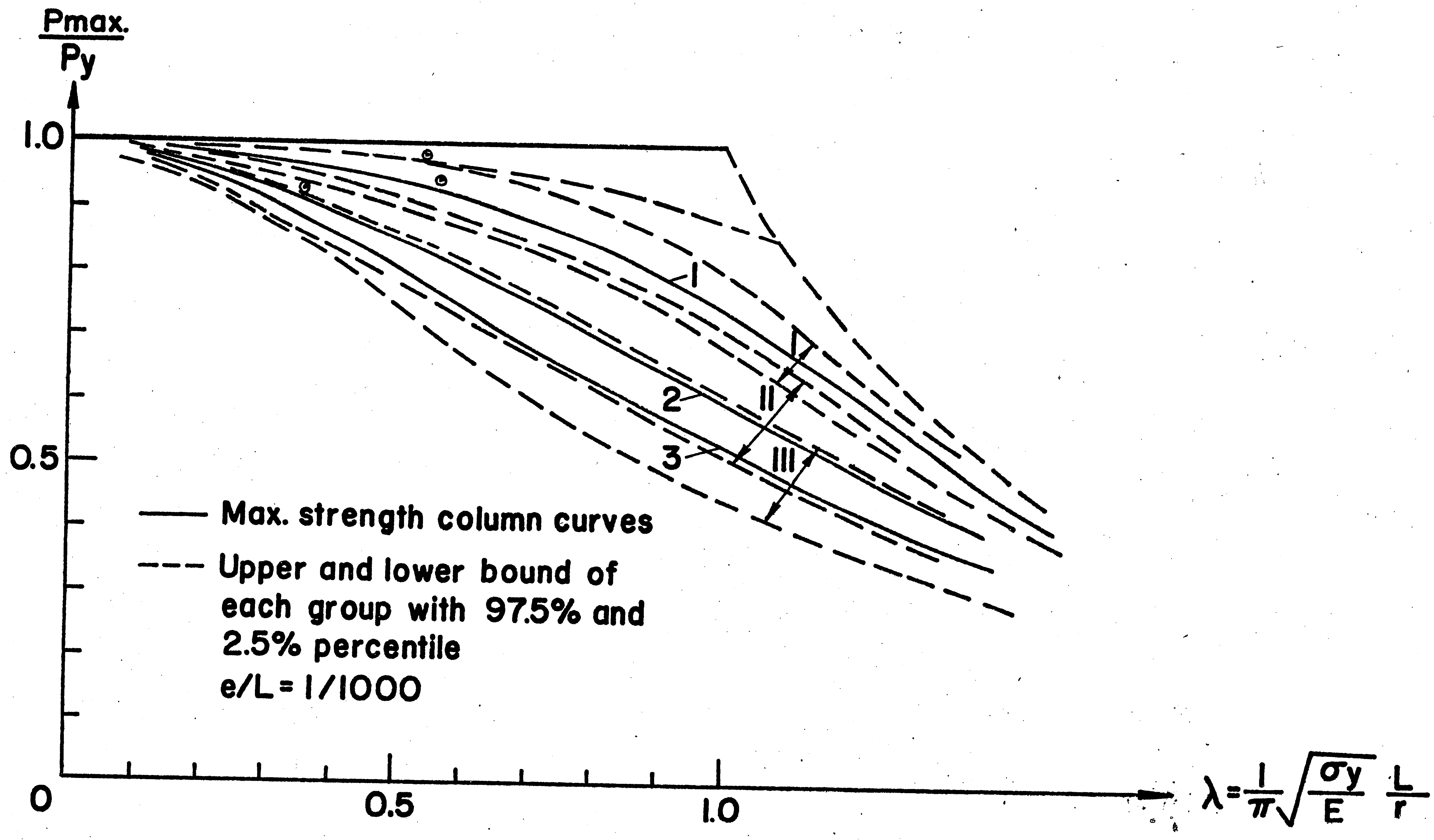

Fig. 3 Test points for rolled box columns

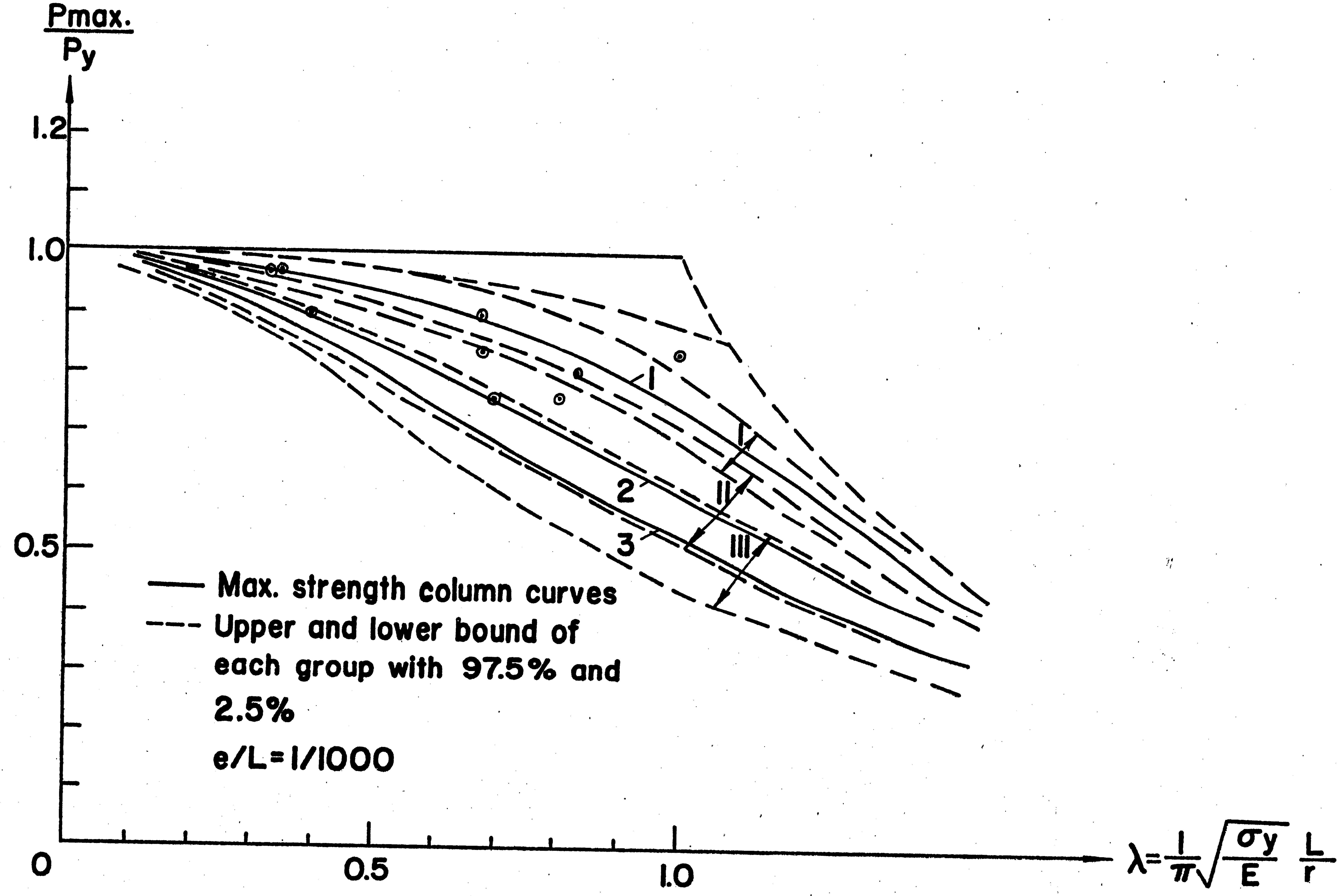

Fig.4 Test points for welded wide flange

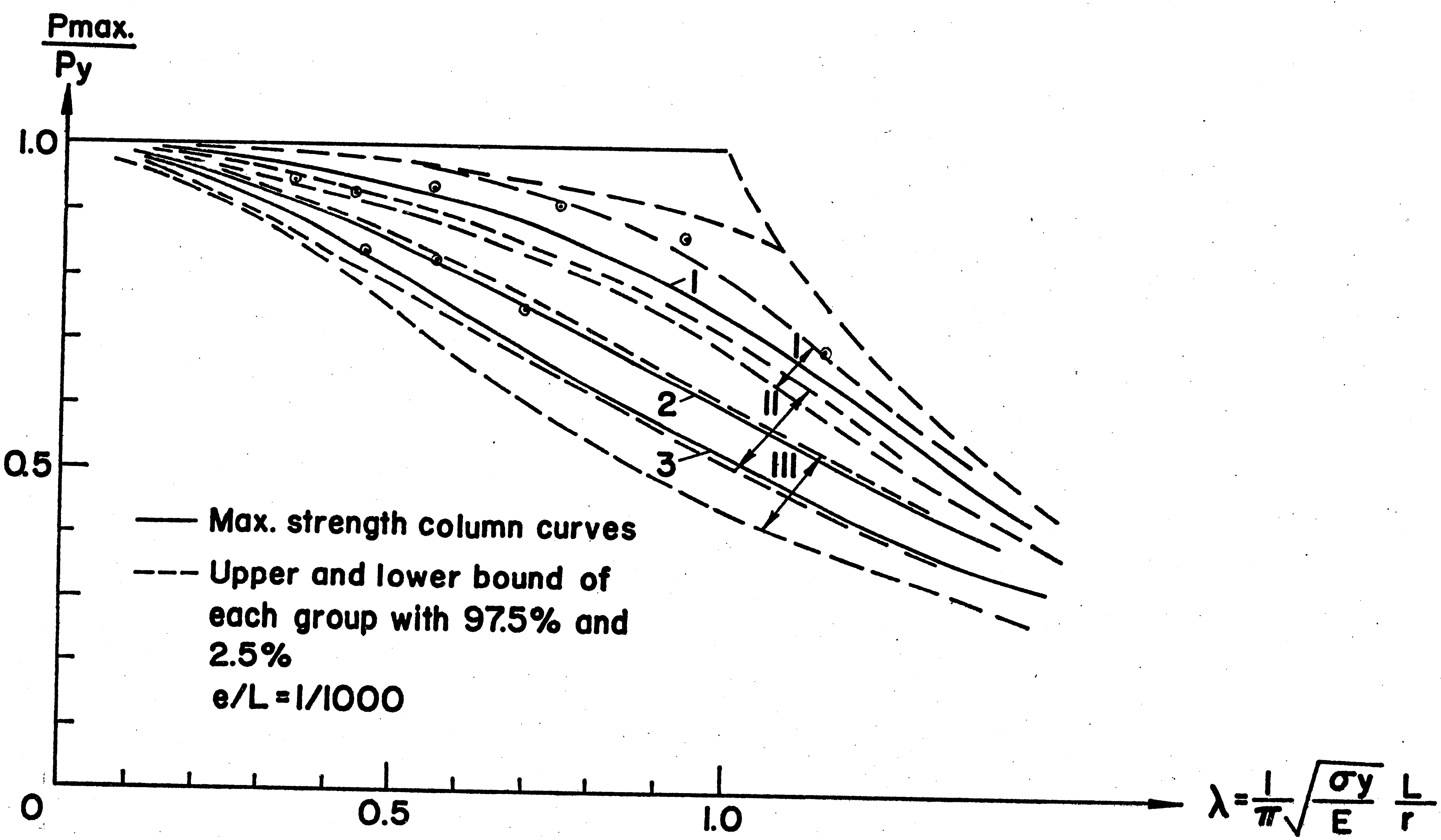

Fig.5 Test points for welded box columns

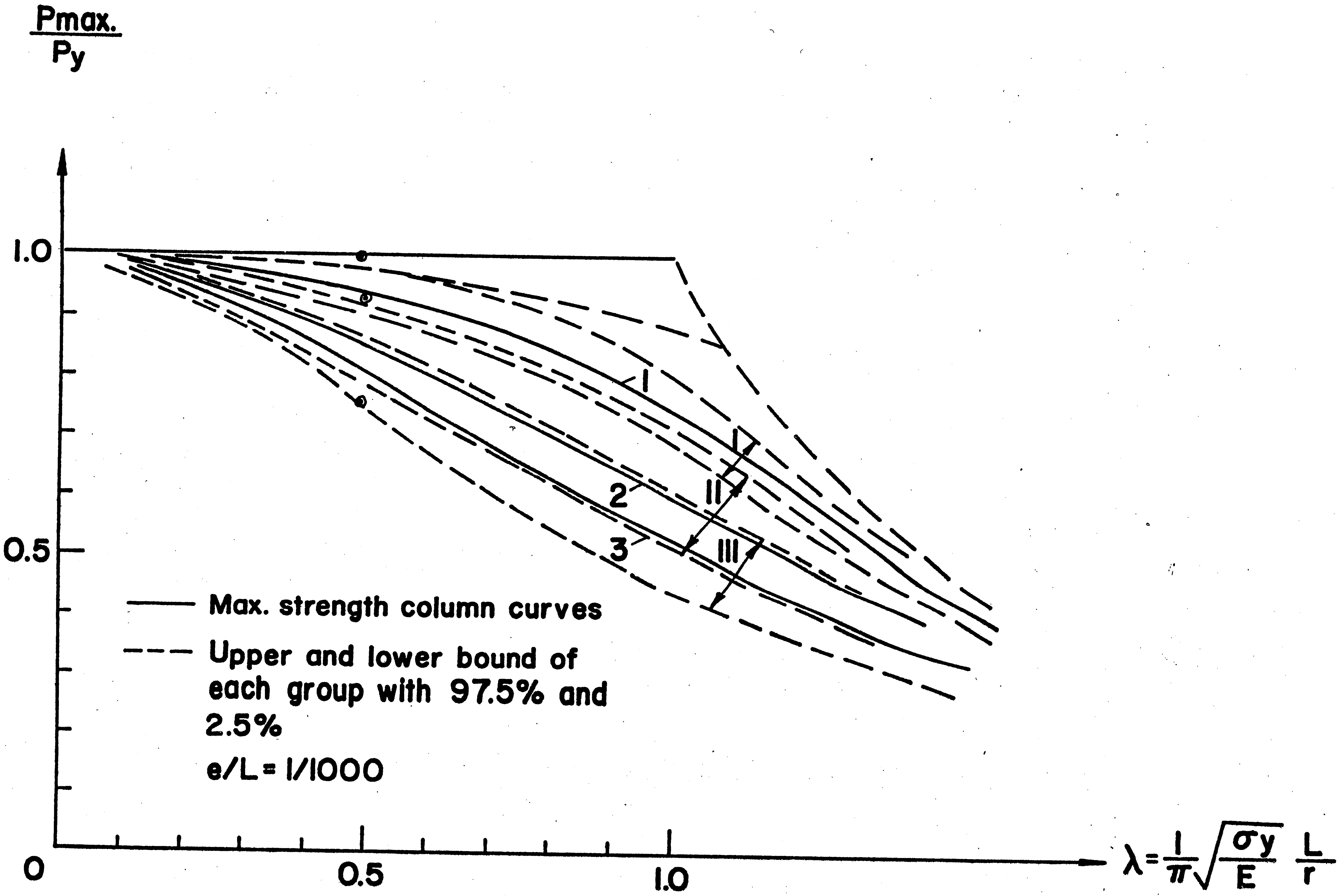

Fig. 6 Test points for japanese columns (σy = 45 ksi)

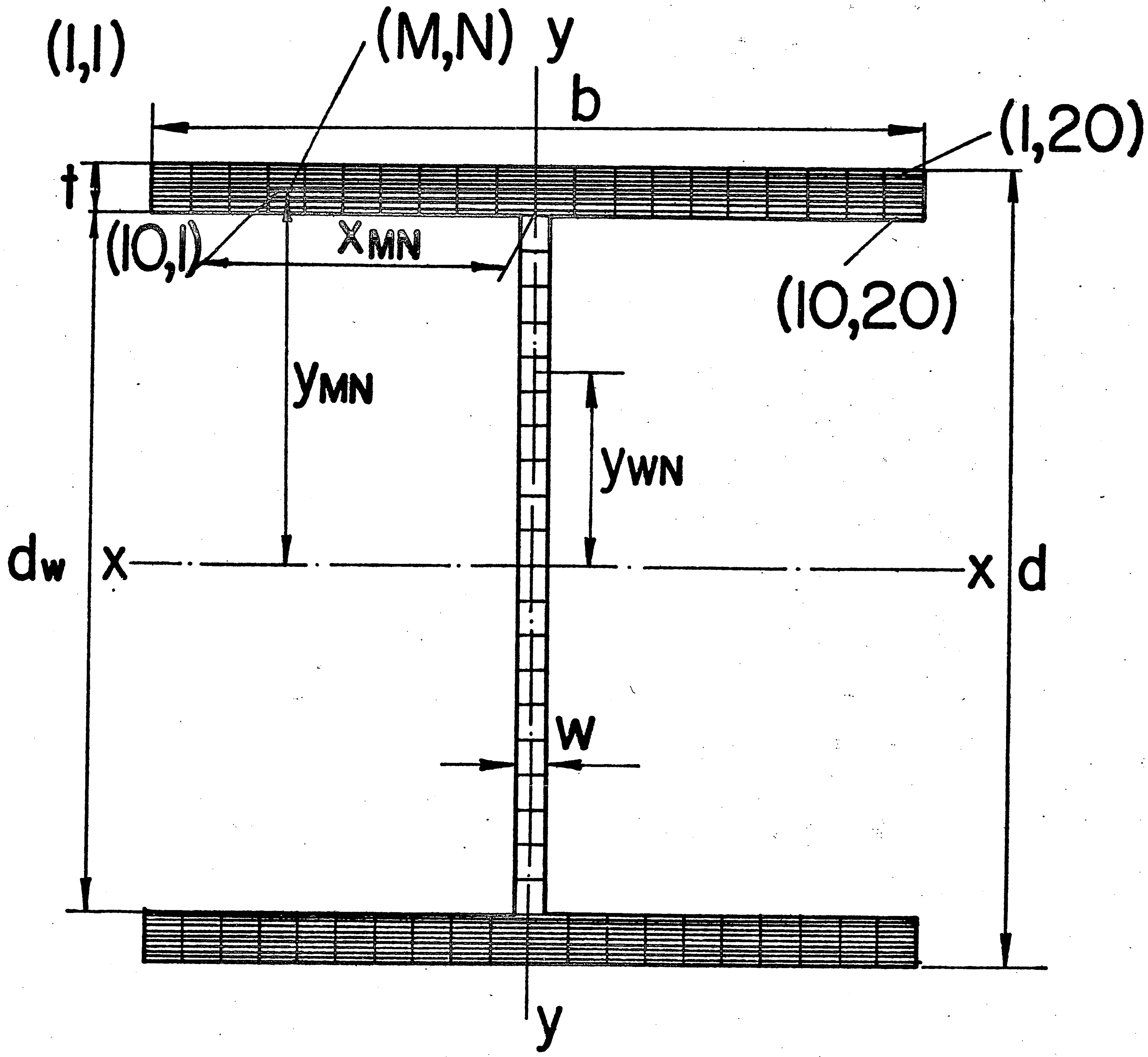

Fig. 7 Arrangement of finite area elements

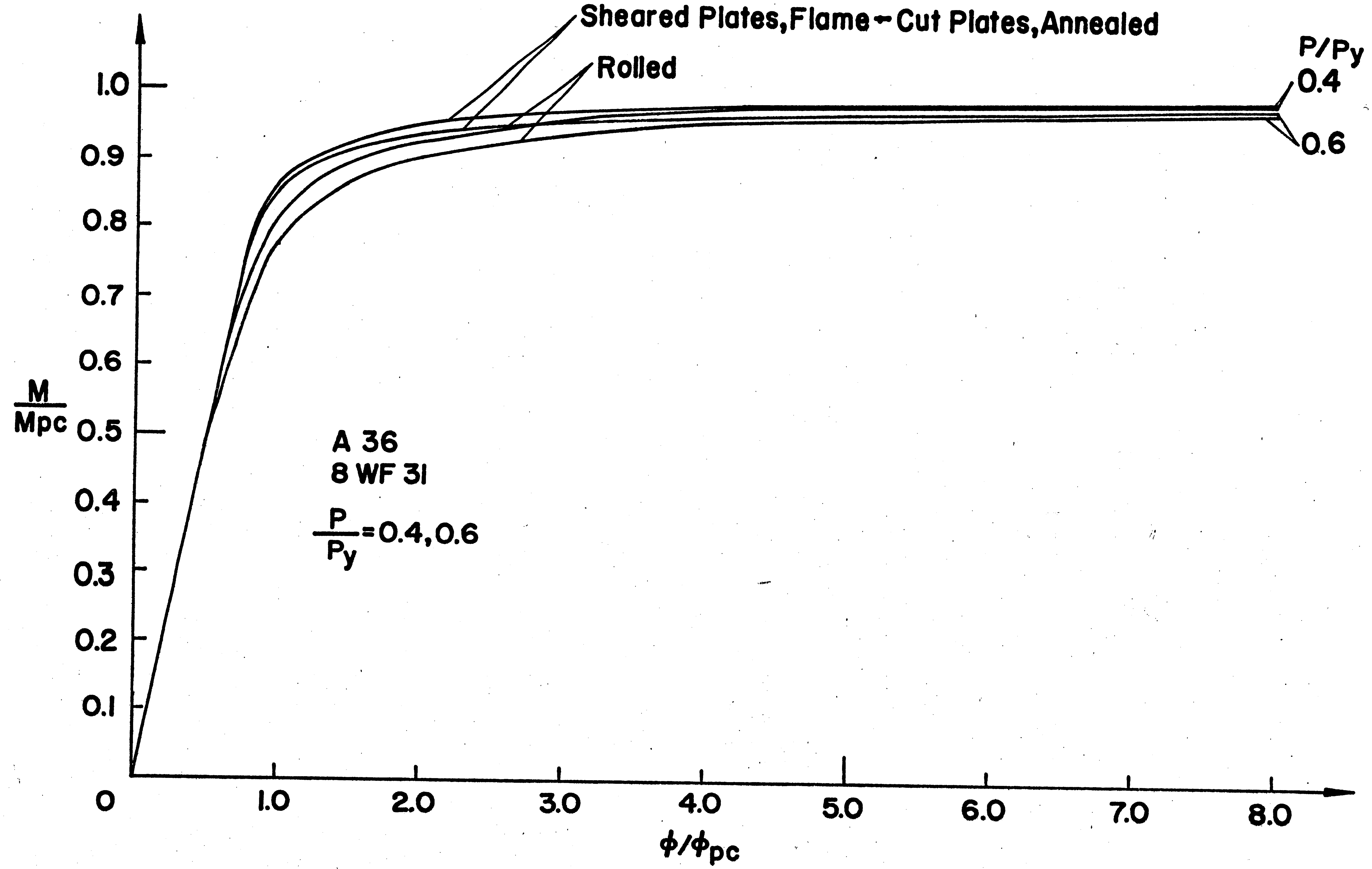

Fig.8 Comparison of Moment-Thrust-Curvature Relationship

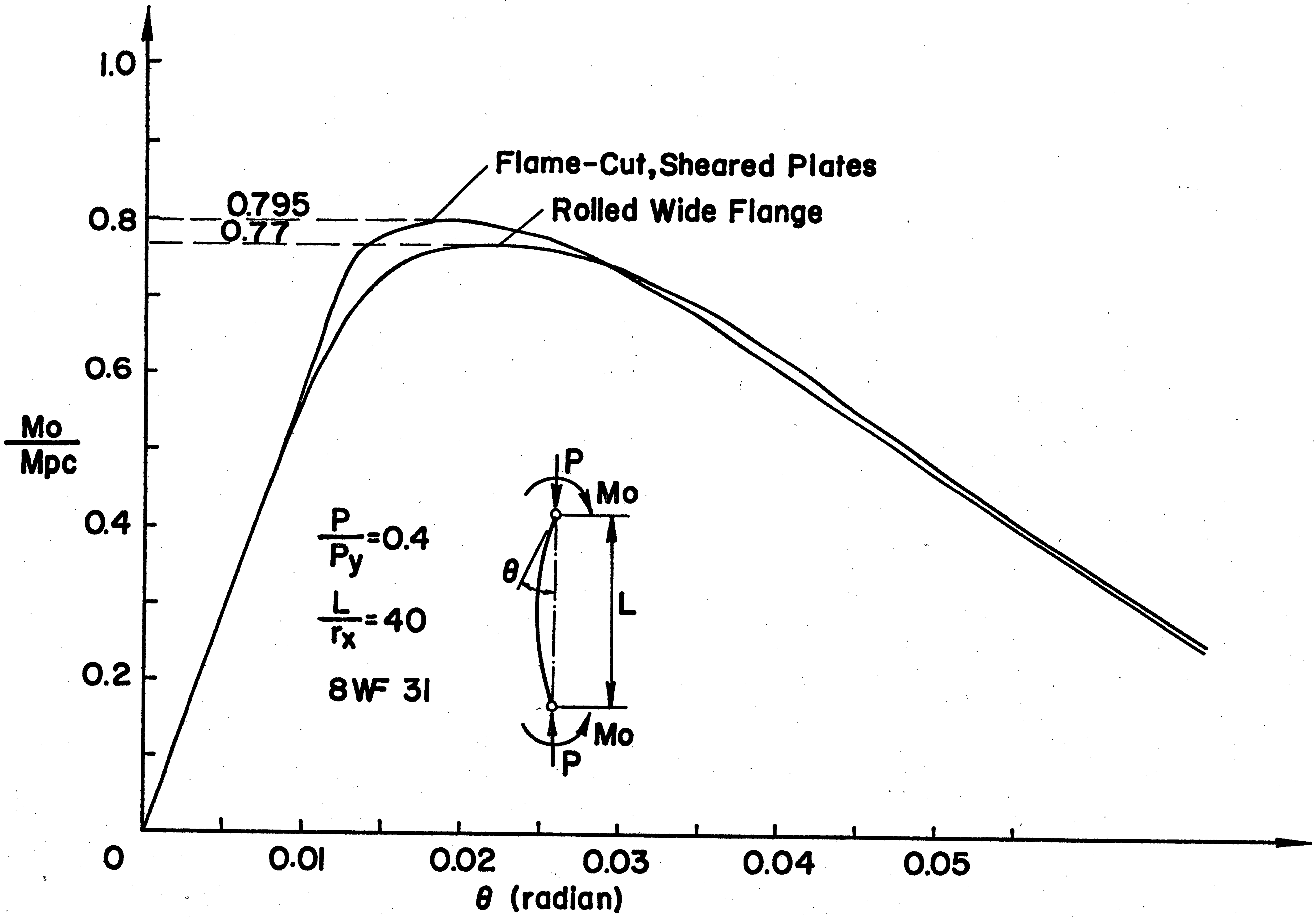

Fig.9 Moment-rotation relationship for beam-columns

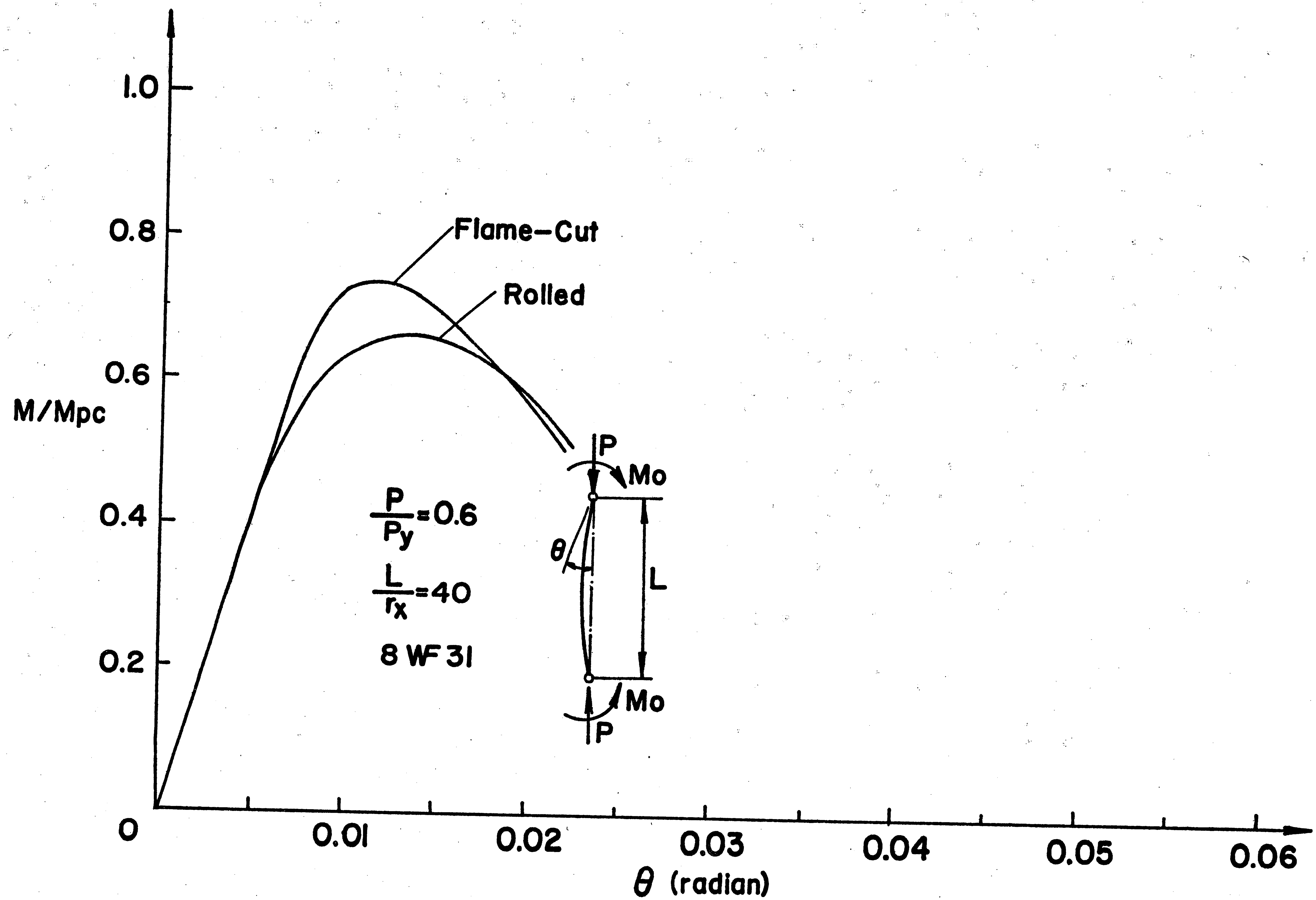

Fig. 10 Moment-end rotation curve for beam-column

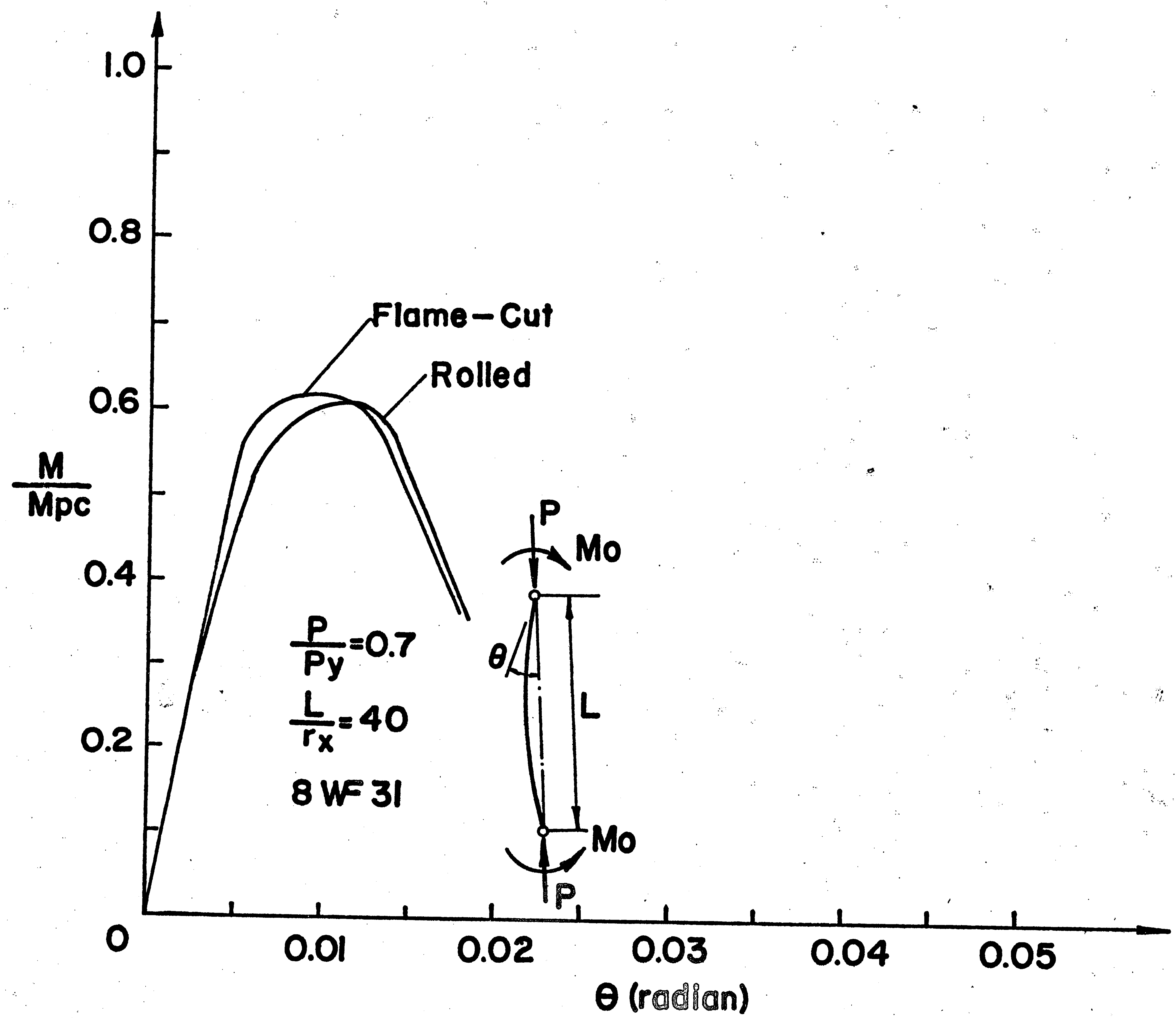

Fig.11 Moment-end rotation curve for beam-column

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9. VITA

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